REGULATING STRUCTURES

- Water which enters into main canal has to be divided into different branches and distributaries as per the demand. The process is called regulation.
- These are the structures which are constructed on a canal to control and regulate the discharge, depth, velocity etc across the canal.

Various structures for canal regulation are:

1) Canal escapes
2) Head regulator and cross regulator
3) Canal Falls
4) Canal outlets and modules

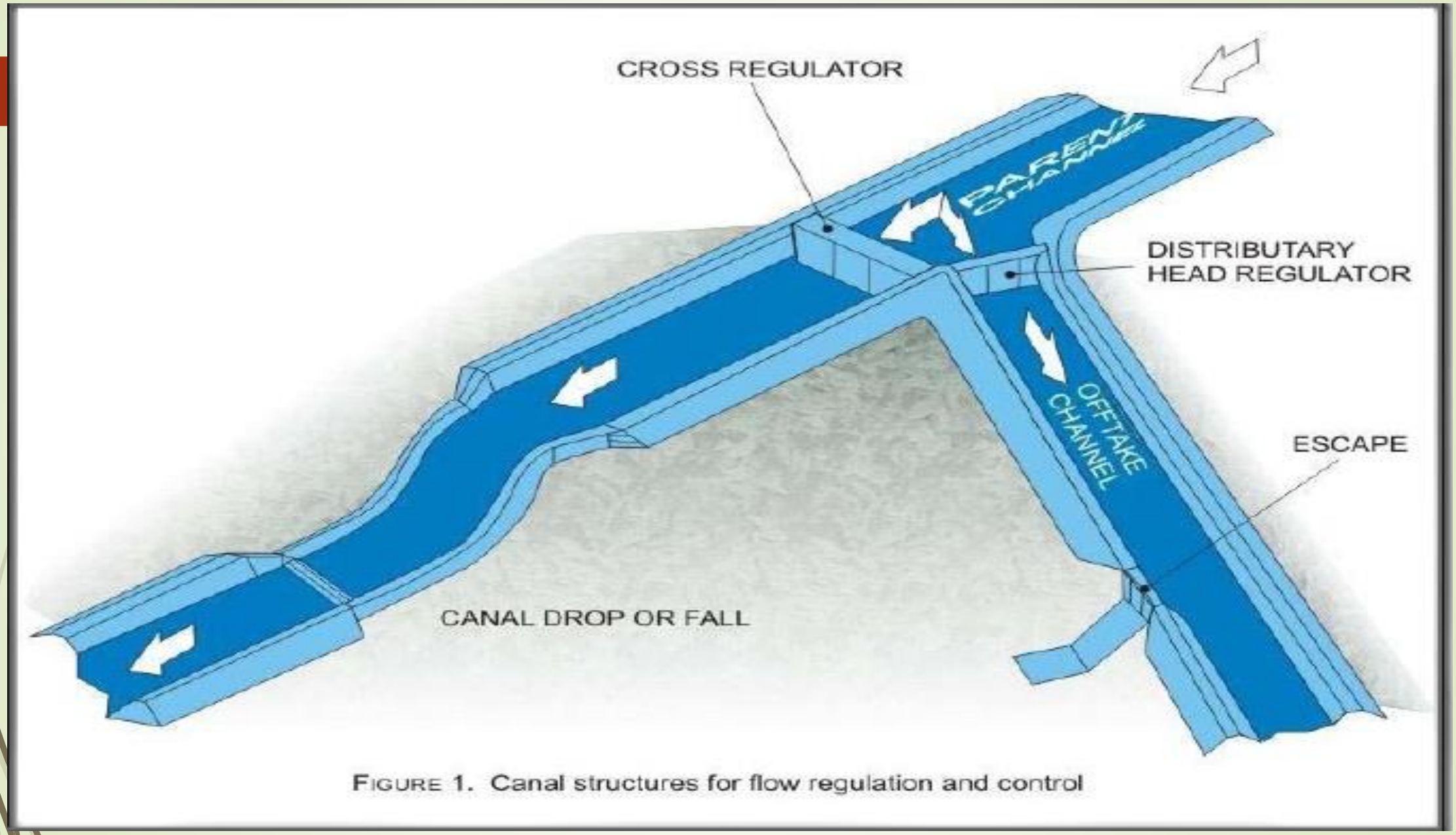


FIGURE 1. Canal structures for flow regulation and control

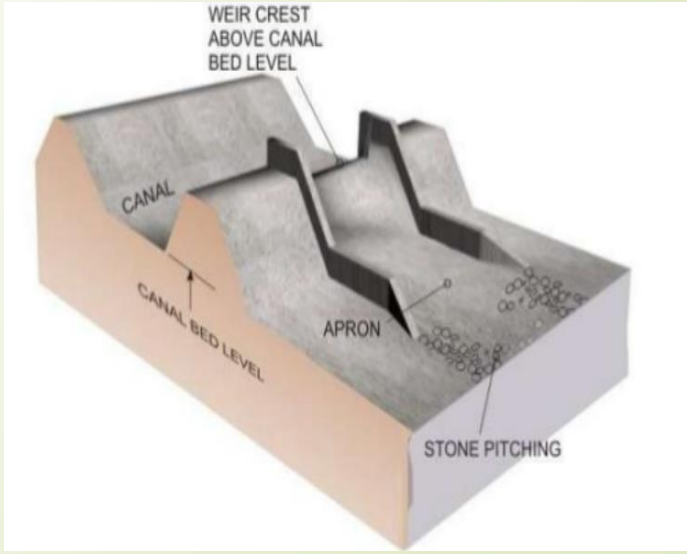
Canal Escape

It is a side channel constructed to remove surplus water from an irrigation channel (main canal, branch canal, or distributary etc.) into a natural drain. The water in the irrigation channel may become surplus due to –

- Difficulty in regulation at the head
- Excessive rainfall in the upper reaches
- Outlets being closed by cultivators as they find the demand of water is over

Function of canal escape

- It helps to overflow the extra surplus water from the canal safely.
- It prevents the damage and de-function of irrigation canal.
- It prevents the damage of the farming land.



❖ Types of canal escape

(a) Weir type escape

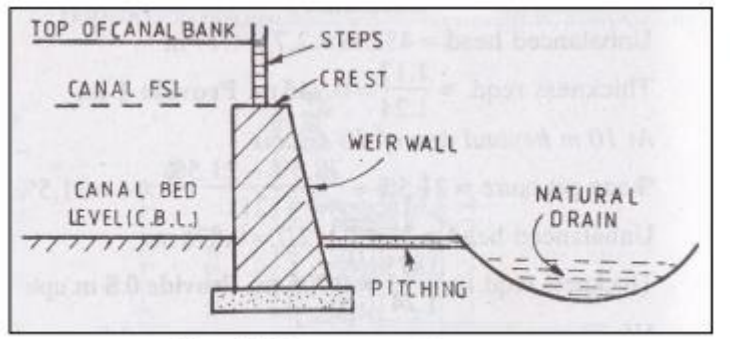


Fig: Weir type canal escape

Crest level = FSL of the canal

Water escapes if w_L > FSL

The crest of the weir wall is kept at R.L equal to canal FSL. When the water level rises above FSL, it gets escaped.

(b) regulator/sluice type escape

The sill of the escape is kept at canal bed level and the flow can be used for completely emptying the canal.

They may be constructed for the purpose of scouring off excess bed silt deposited in the head reaches from time to time.

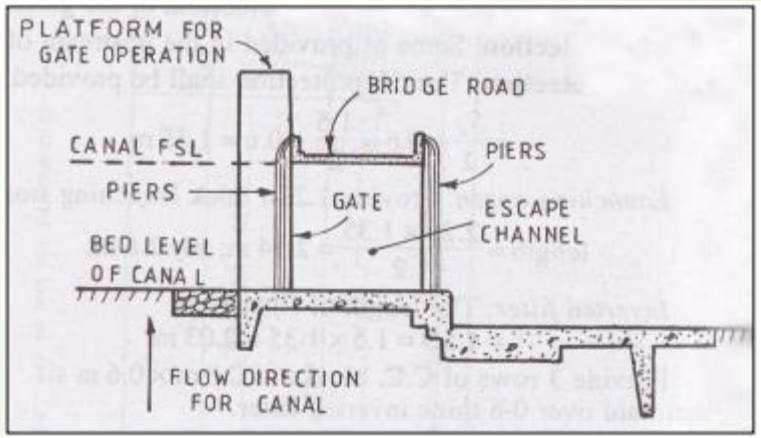


Fig: Regulator type canal escape

Crest

Made with KINEMAST

❖ Alignment of off-taking canals

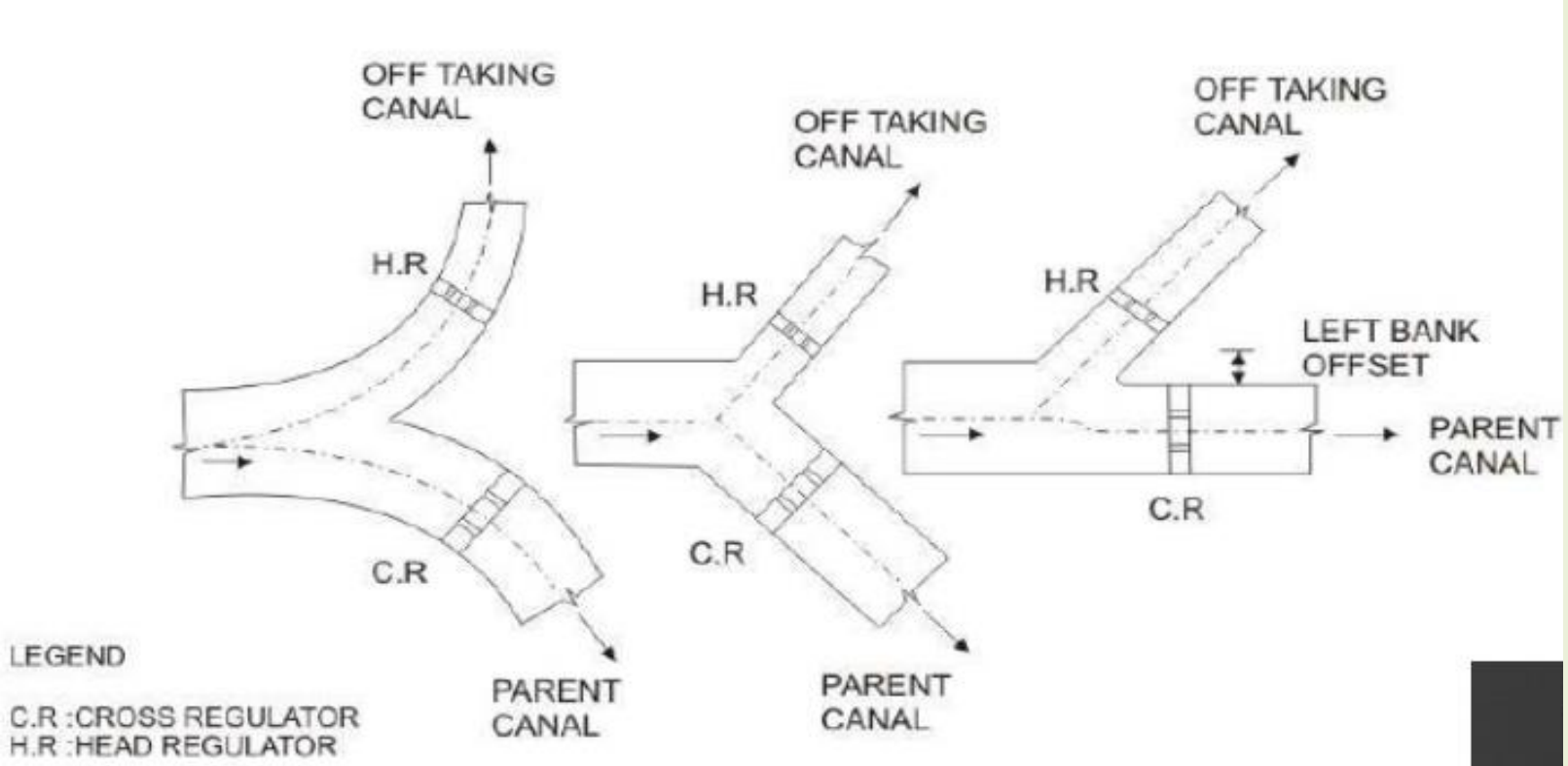


FIGURE 20. Alignment types for off taking canal from a parent canal
(a) Smooth off take
(b) Both inclined to original flow;
(c) Parent canal flows straight with reduced width.

❖ Cross Regulator

It is a hydraulic structure constructed across the canal to regulate the irrigation water supplies. It may be constructed across any type of canal main, branch or a distributary.

1. To effectively controls the canal irrigation system.
2. To raise the water level in the parent channel on the upstream so that off taking canal can takes its full supply even when the water level in the parent channel is lower than F.S.L.
3. They help in absorbing fluctuations in various sections of canal system and help in preventing possibilities of any breach.
4. There is usually a bridge on the cross regulator, which provides a means of transportation.

❖ Head regulator and cross regulator

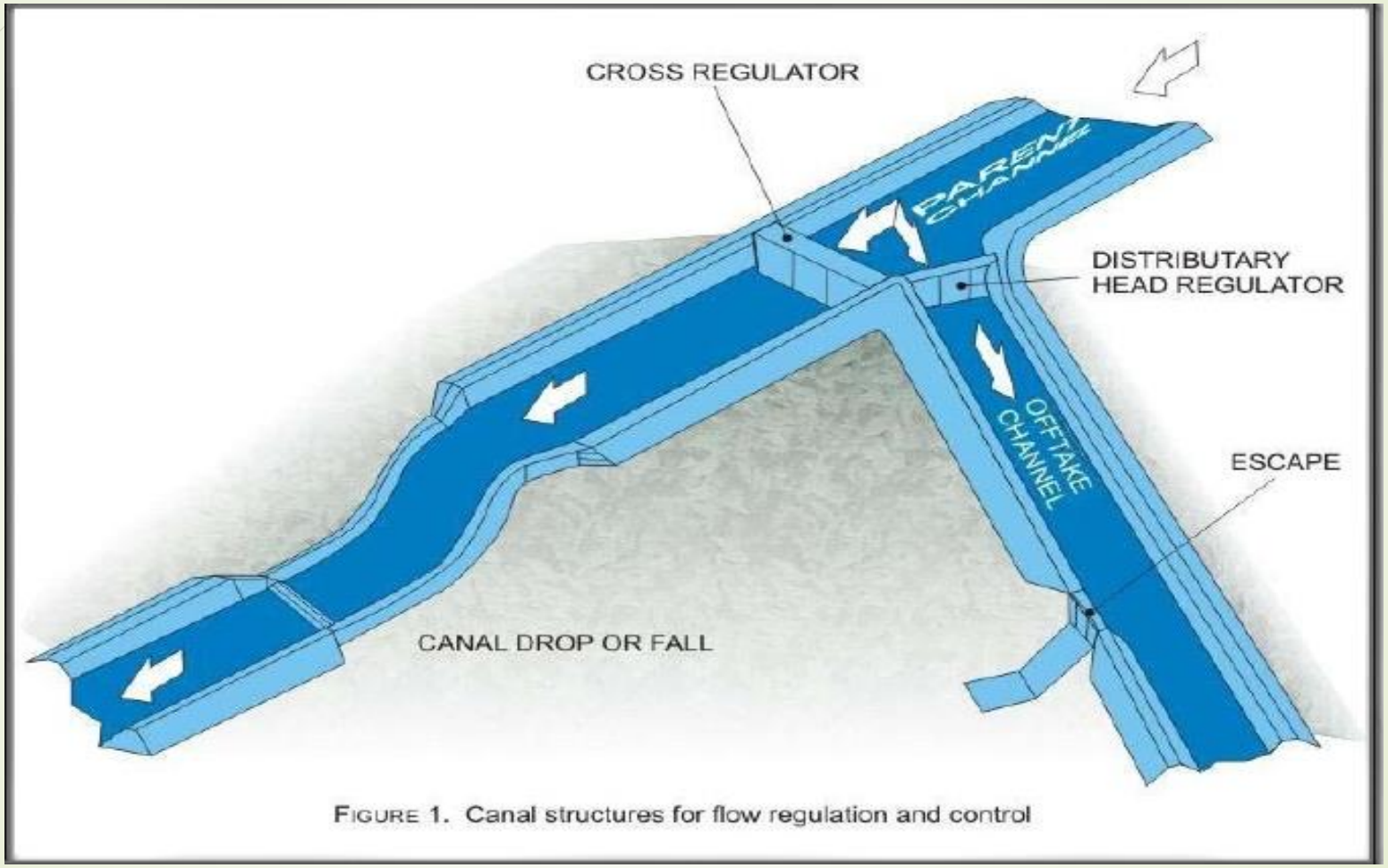
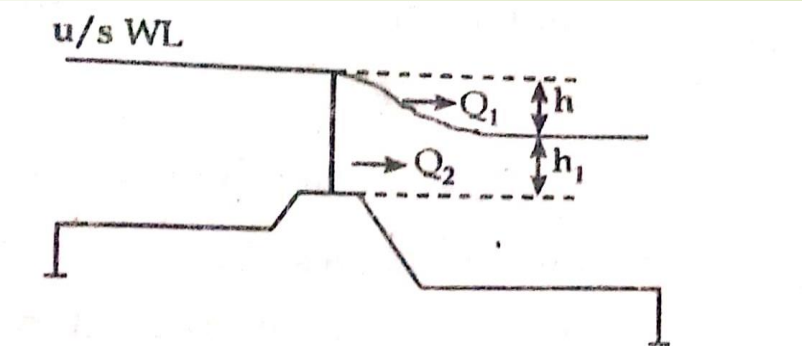


FIGURE 1. Canal structures for flow regulation and control

❖ Distributary head regulator

A distributary head regulator controls the supply of the off taking canals while a cross regulator controls the supply of parent channels.

1. It regulates the supply of water from parent channels to off taking channel.
2. It controls the entry of silt in to the off taking channel.
3. It can serve as the meter for measurement of discharge.
4. It used for the shutting of the supply of the off-taking channel when water is not needed.



HEAD REGULATOR
CROSS REGULATO

Q.N. Design a cross regulator and a head regulator for a channel which takes off from the parent channel with following data.

Discharge of parent channel = 150 cumecs

Discharge of distributary = 16 cumecs

F.S.L. of the parent channel, u/s = 210.0 m

F.S.L. of the parent channel, d/s = 209.8 m

Bed width of parent channel, u/s = 52 m

Bed width of parent channel, d/s = 46 m

Depth of water in parent channel d/s and u/s = 2.5 m

F.S.L. of distributary = 109.1 m

Silt factor = 0.8 m

Assume safe exit gradient = $\frac{1}{5}$

Solution:

- *Design of cross regulator*
- **Crest level**
- Crest level of cross regulator kept same as u/s bed level of parent channel = 210.0 -2.5 = 207.5 m
- Provide crest level at R.L. 207.5 m
- Waterway, neglecting velocity head
- Q = B $\sqrt{h}$ [1.69 h + 3.54 h_1]

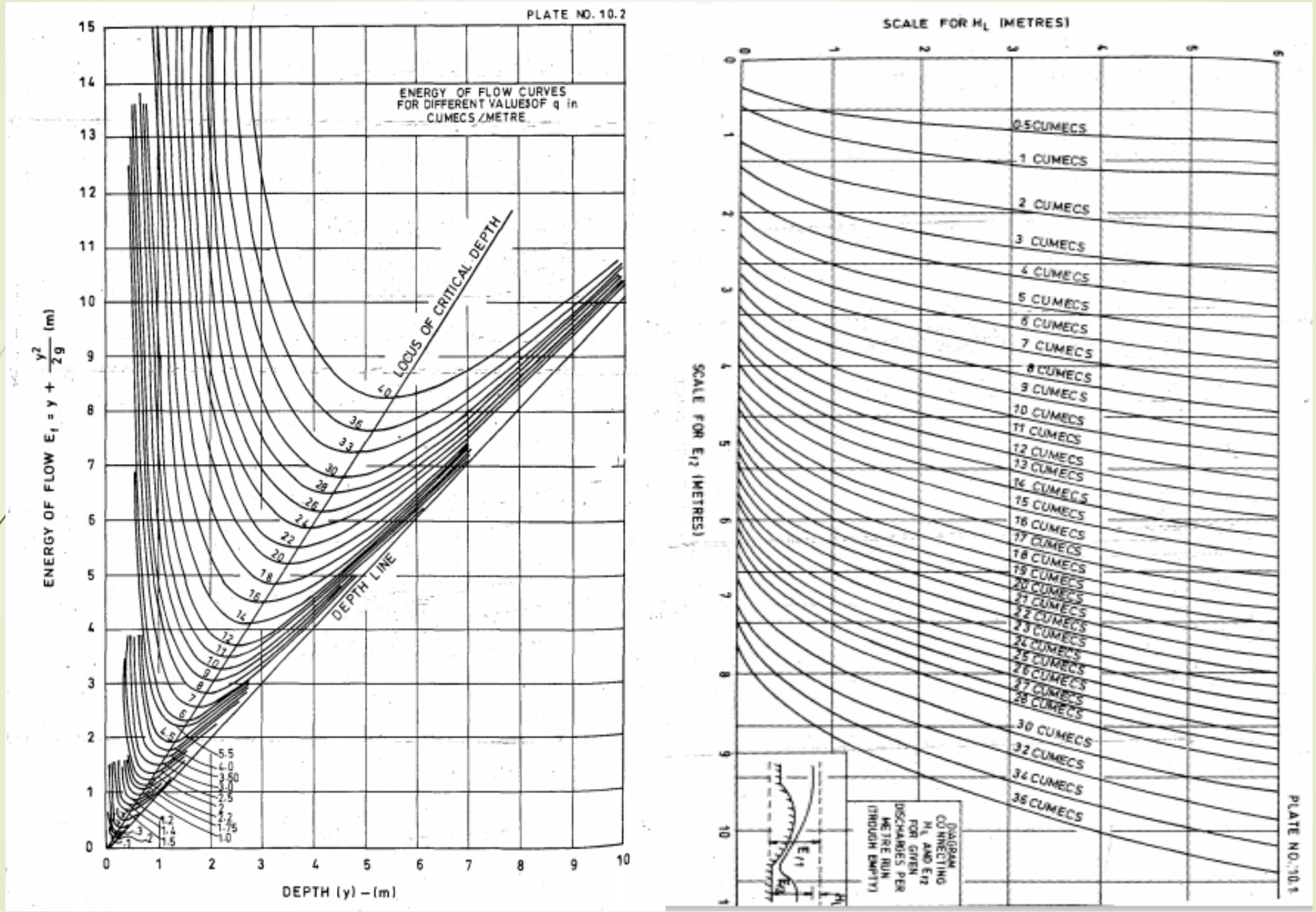


PLATE NO. 10.2
ENERGY OF FLOW CURVES FOR DIFFERENT VALUES OF q in CUMECS / METRE
LOCUS OF CRITICAL DEPTH
DEPTH LINE
ENERGY OF FLOW $E_1 = y + \frac{y^2}{2g}$ (m)
DEPTH (y) – (m)
SCALE FOR H_L (METRES)
SCALE FOR E_{12} (METRES)
0.5 CUMECS
1 CUMECS
2 CUMECS
3 CUMECS
4 CUMECS
5 CUMECS
6 CUMECS
7 CUMECS
8 CUMECS
9 CUMECS
10 CUMECS
11 CUMECS
12 CUMECS
13 CUMECS
14 CUMECS
15 CUMECS
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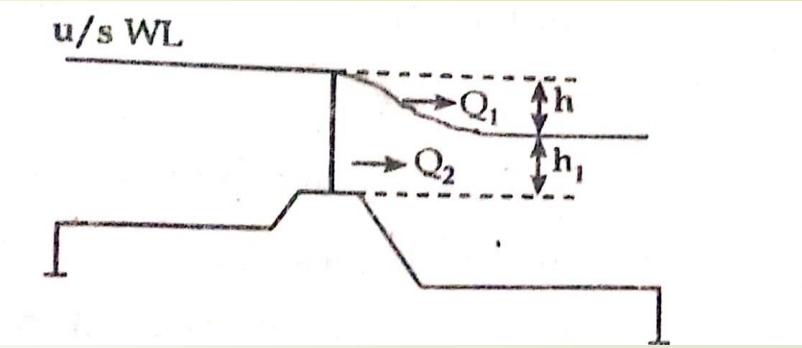
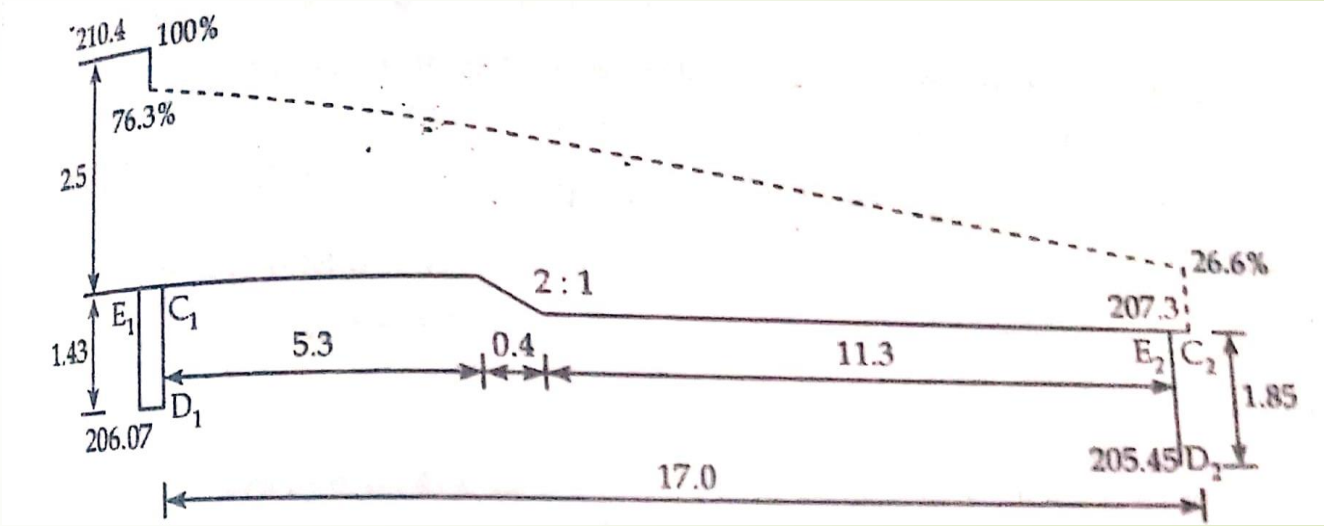
25 CUMECS
26 CUMECS
27 CUMECS
28 CUMECS
30 CUMECS
32 CUMECS
34 CUMECS
36 CUMECS
DIAGRAM CONNECTING H_L AND E_{12} FOR GIVEN DISCHARGES PER METRE RUN (THROUGH EMPTY)
PLATE NO. 10.1

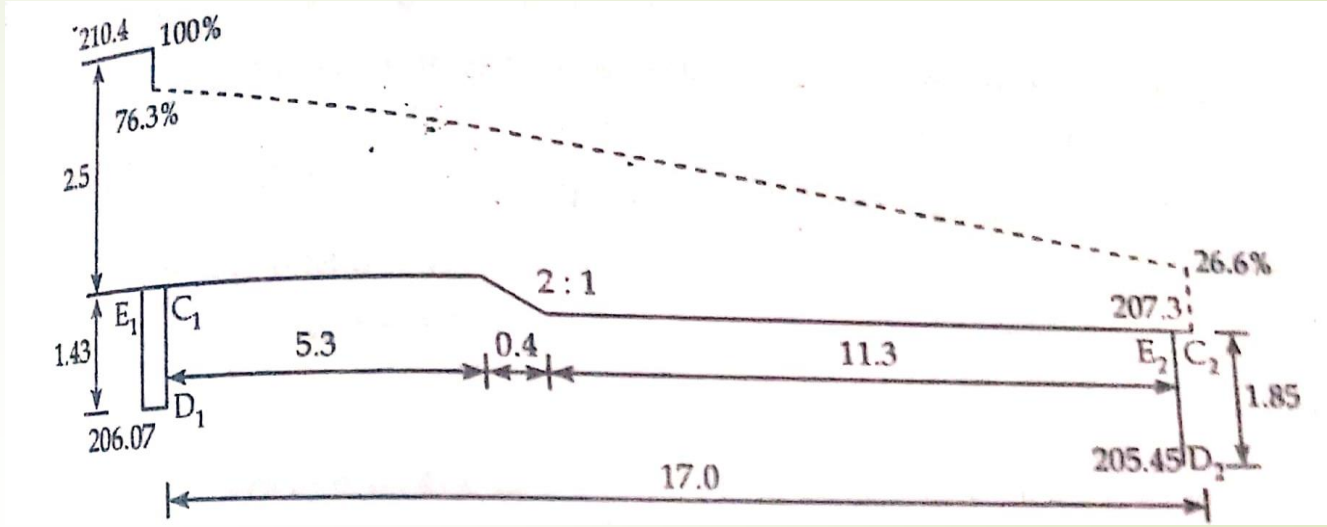
- Where, B is the clear waterway required.
- In this case,
- h = u/s F.S.L. – d/s F.S.L. = 210.0 – 209.8 = 0.2 m
- h_1 = d/s F.S.L. – crest level = 209.8 – 207.5 = 2.3 m
- 150 = B $\sqrt{0.20}$ [1.69*0.2 + 3.54*2.3]
- B = 39.53 m
- Provide 5 bays of 8.0 m each with a clear waterway = 5*8.0 = 40.0 m
- Provide 4 piers of 1.5 m width each = 6.0 m
- Thus,
- Overall waterway = 40 + 6 = 46 m
- **Downstream floor level or cistern level**
- q = $\frac{150}{40}$ = 3.75 cumecs / meters
- H_L = 0.2 m
- From figure 5.27;
- E_{f2} = 1.85 m
- d/s floor level = d/s F.S.L. - E_{f2} = 209.8 – 1.85 = 207.95 m
- d/s bed level = 209.8 – 2.5 = 207.3 m
- hence provide the cistern or d/s at R.L. 207.3 m





Length of d/s floor

- Length of d/s floor required = 5 (y_2 - y_1)
- From figure 5.27;
- for E_{f2} = 1.85 m;
- y_2 = 1.8 m
- For E_{f1} = E_{f2} - H_1= 1.85 - 0.2 =1.65 m;
- y_1= 0.8 m
- Therefore, Length of d/s floor required = 5 (y_2 - y_1) = 5(1.8-0.8) = 5.0 m



U/S cut-offs

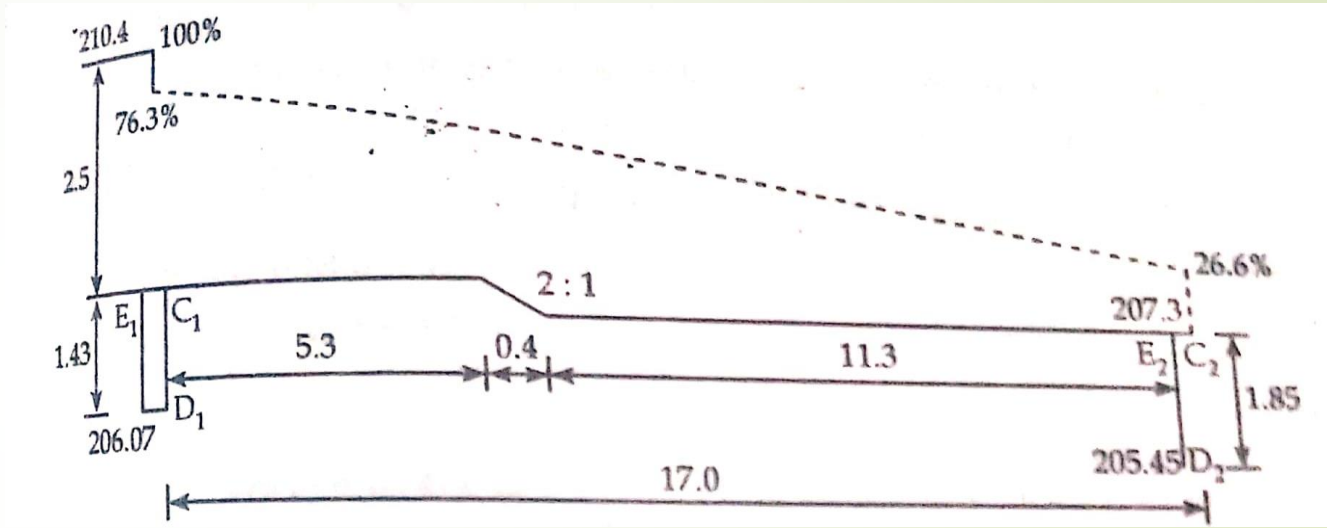
- Provide upstream cut-off for a depth = $\frac{y_u}{3}$ + 0.6 m = $\frac{2.5}{3}$ + 0.6 = 1.43 m
- Below the floor,
- i.e, level of its bottom = 207.5 – 1.43 = 206.07 m

Downstream cut-offs

- Depth od d/s cutoff below floor level = $\frac{y_d}{2}$ + 0.6 m = $\frac{2.5}{2}$ + 0.6 = 1.85 m
- Hence,
- Bottom level of d/s cutoff = 207.30 – 1.85 = 205.45 m

Total floor length from exit gradient considerations;

- $G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$
- Here, H = Maximum static head which is caused when there is full water on u/s and there is no water on d/s.
- H = u/s F.S.L. – d/s bed level = 210.0 – 207.3 = 2.7 m
- d = Depth of d/s cutoff = 1.85 m
- $G_E = \frac{1}{5}$ (Given)
- $\frac{1}{5} = \frac{2.7}{1.85}\frac{1}{\pi\sqrt{\lambda}}$
- $\frac{1}{\pi\sqrt{\lambda}} = 0.137$
- From plate 11.2;
- For $\frac{1}{\pi\sqrt{\lambda}} = 0.137$;
- α = 9



- Therefore, b = α d = 9*1.85 = 16.65 m (say 17 m; for more safe)
- Minimum d/s floor length required = $\frac{2}{3}$ b = $\frac{2}{3}$ *17 = 11.3 m
- Which is greater than 5.0 m i.e; 5 (y_2 - y_1)
- Hence, provide 11.3 m as the d/s floor length.
- Glacis length = 2*0.2 = 0.4 m
- The balance i.e, 17-11.3 -0.4 = 5.3 m is provided as u/s floor length, as shown in figure.

- Calculation of uplift pressure and floor thickness (Home work)
- Design of distributary head regulator (Home work)

Canal Falls (Canal Drops)

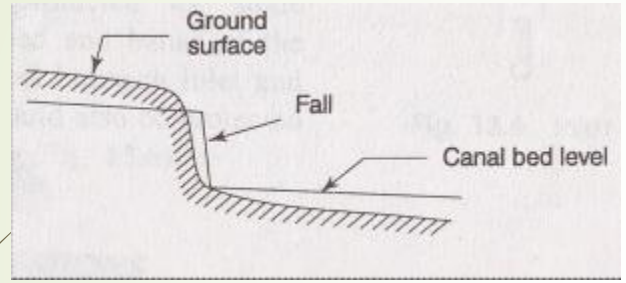
Canal Falls (Canal Drops)

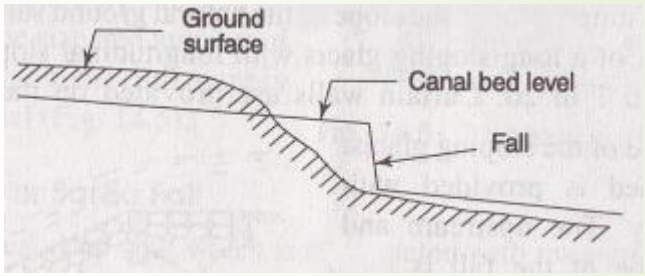
Irrigation canals are constructed with some permissible bed slopes so that there is no silting or scouring in the canal bed. But it is not always possible to run the canal at the desired bed slope throughout the alignment due to the fluctuating nature of the country slope.

Whenever available natural ground slope is steeper than designed bed slope of the channel, the difference is adjusted by constructing structures in the canal called canal fall or canal drop structure.

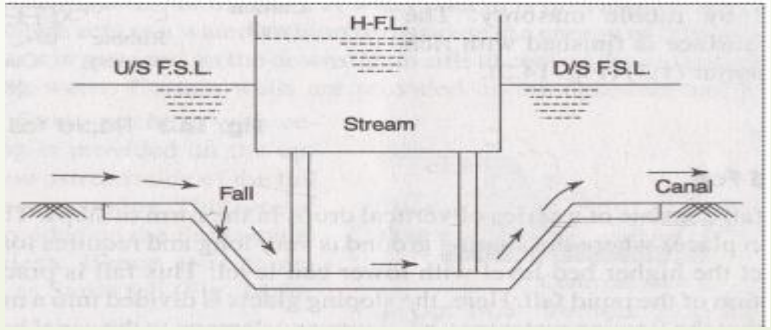
Necessity / Importance of Canal Falls:

- When the slope of the ground suddenly changes to steeper slope, the permissible bed slope can not be maintained. It requires excessive earthwork in filling to maintain the slope. In such a case falls are provided to avoid excessive earth work in filling.





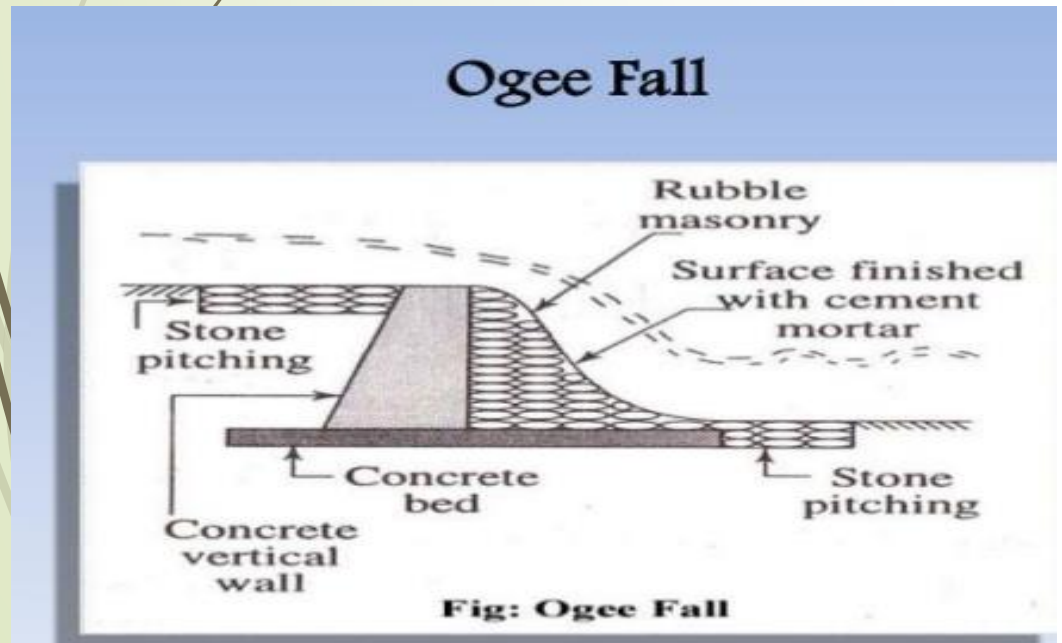
- When the slope of the ground is more or less uniform and the slope is greater than the permissible bed slope of canal. In that case also the canal falls are necessary.
- In cross-drainage works, when the difference between bed level of canal and that of drainage is small or when the F.S.L of the canal is above the bed level of drainage then the canal fall is necessary to carry the canal water below the stream or drainage.



❖ Types of Falls

1. Ogee fall

- An ogee curve (a combination of convex and concave curve) is provided for carrying canal water from higher level to lower level.
- Limited to low depths
- Provides smooth transition from u/s to d/s
- Energy dissipation is low
- Due to smooth transition, kinetic energy is preserved i.e. velocity of flow is not reduced hence it may cause erosion in downstream.



2. Rapid Fall

- Suitable when the slope of natural ground surface is even and long.
- Consists of gentle longitudinal slope which varies from 1 in 10 to 1 in 20.
- Constructed with boulder facing
- Cost of construction is high.

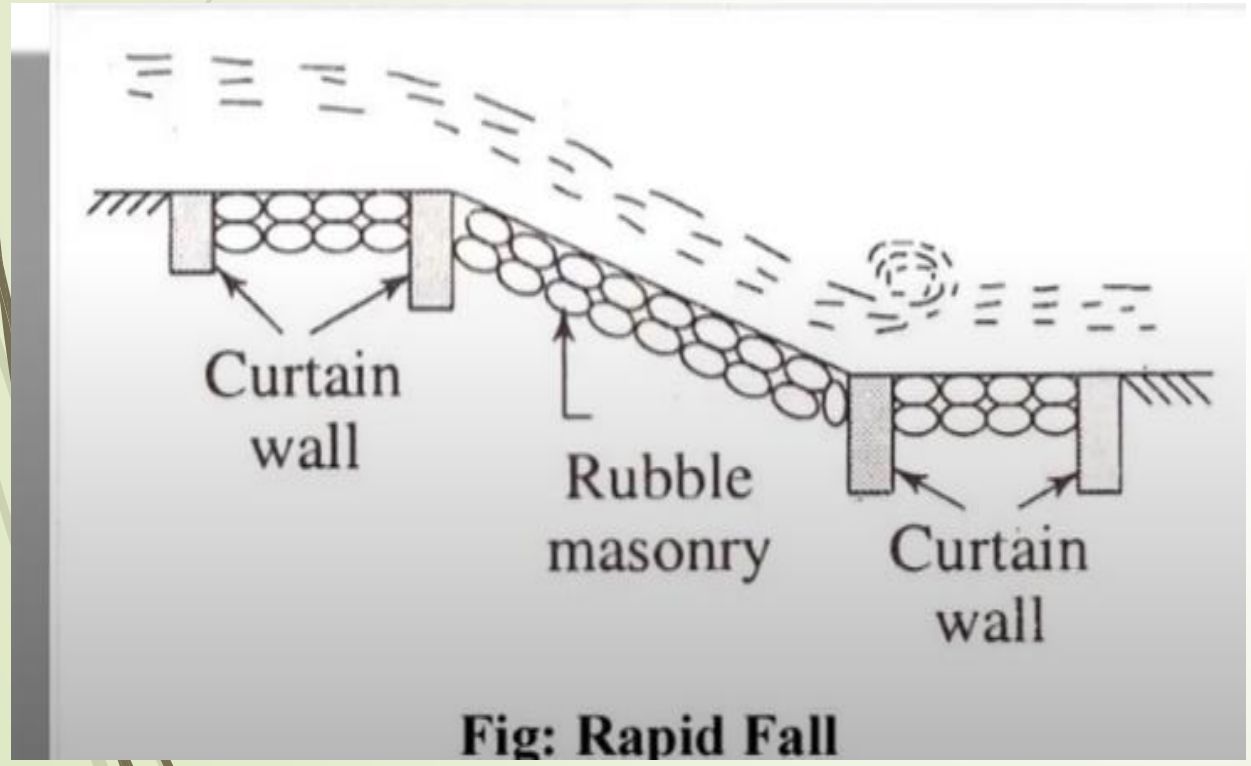
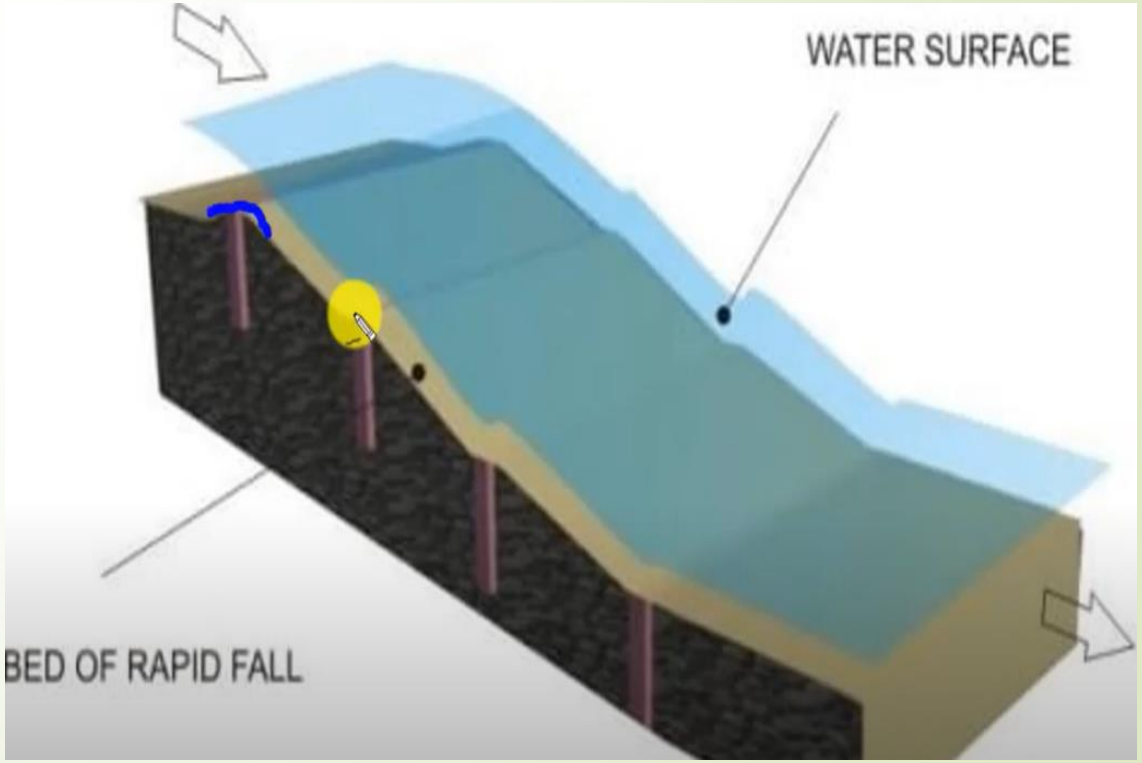


Fig: Rapid Fall



3. Stepped fall

- Modified form of rapid fall.
- Suitable where sloping ground is very long.
- It consists of series of vertical drops in the form of steps.
- Energy dissipation in series of steps.
- Cost is high.

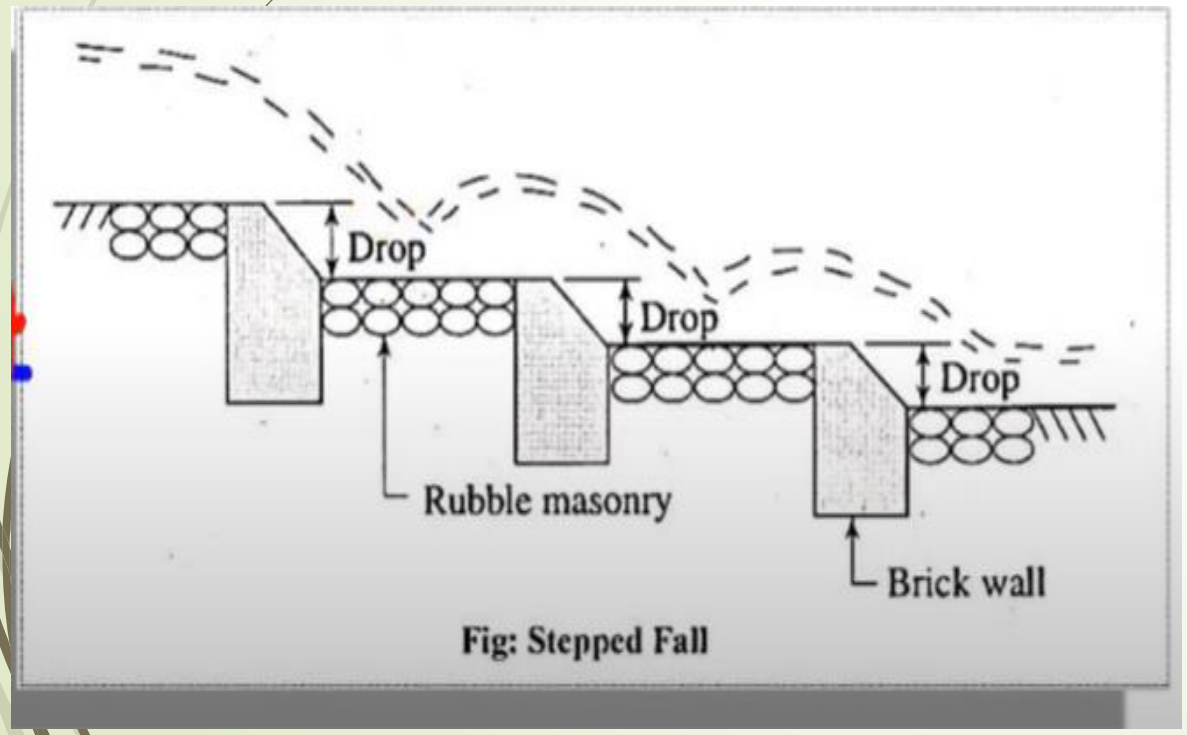


Fig: Stepped Fall

4. Trapezoidal Notch

- In this type the body wall consists of several trapezoidal notches between piers.
- No. of openings may be one or more as required.
- They are common and economical.

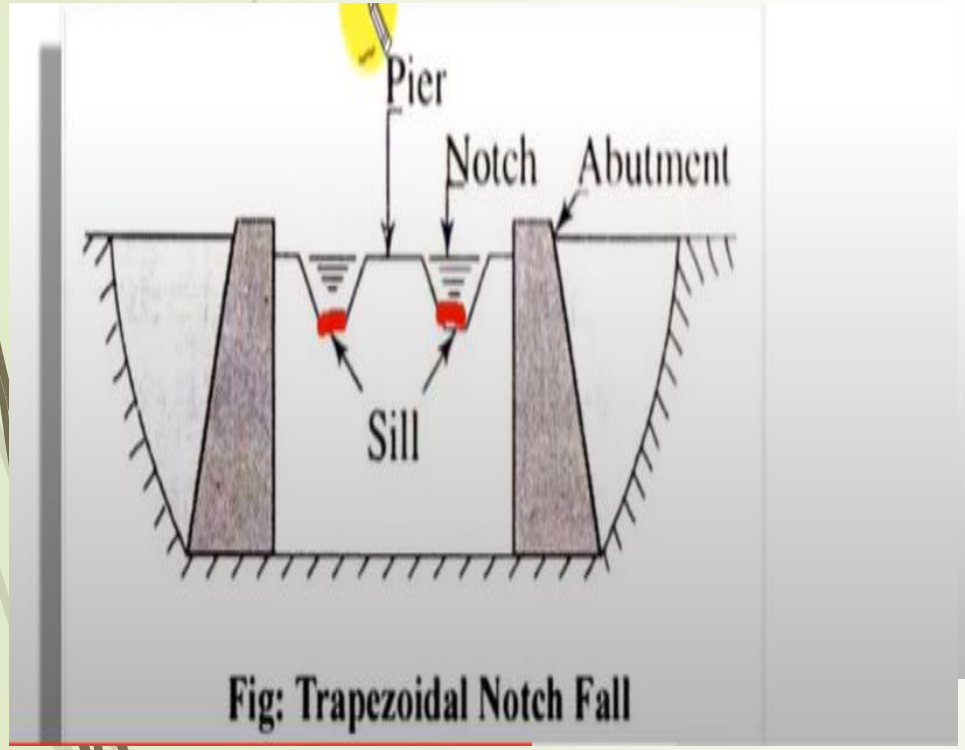
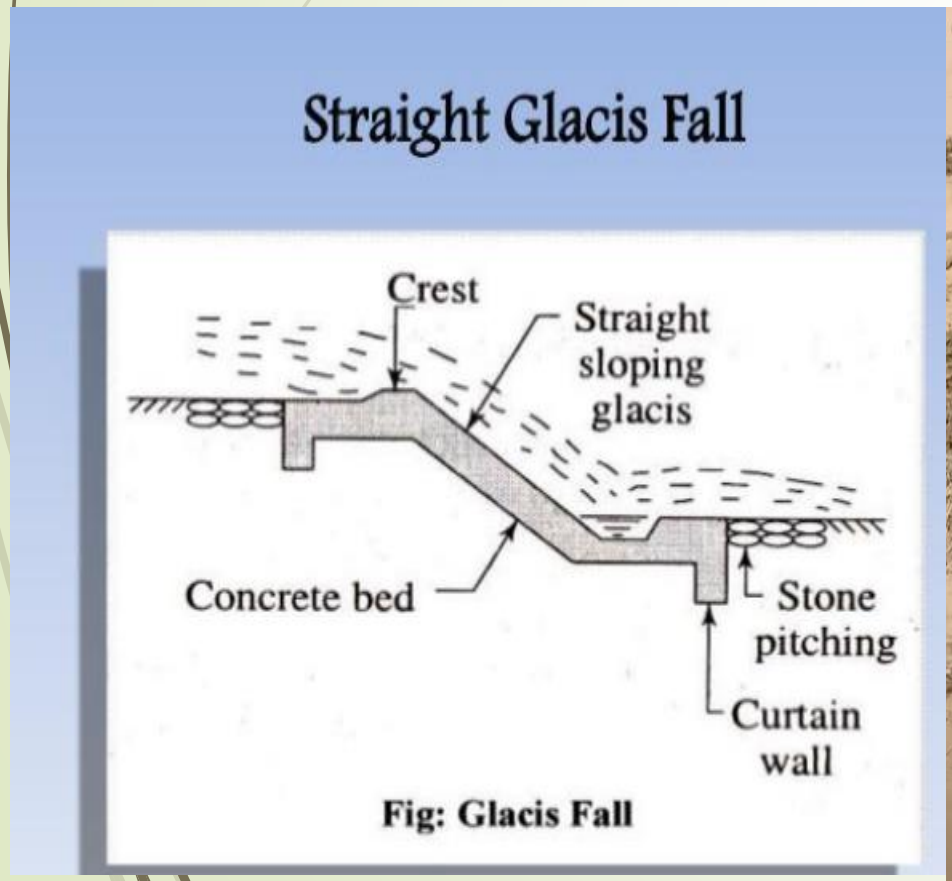


Fig: Trapezoidal Notch Fall

5. Straight glacis fall

- It consists of straight glacis provided with crest wall.
- Utilizes hydraulic jump for energy dissipation.
- Suitable for high discharge upto 60 cumecs and fall upto 1.5 m.



6. Vertical drop fall or Sarda type fall

- A cistern is formed to act as water cushion.
- Easy to construct and economical.
- No clear hydraulic jump is formed and energy dissipation is done turbulent diffusion, as high velocity jet enters the deep pool of water D/s.

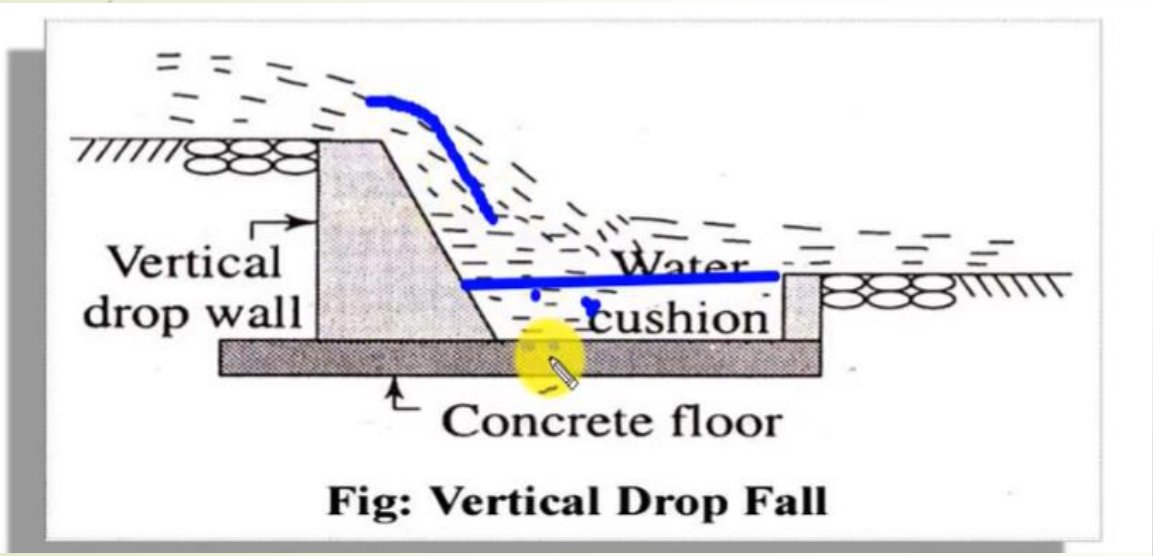
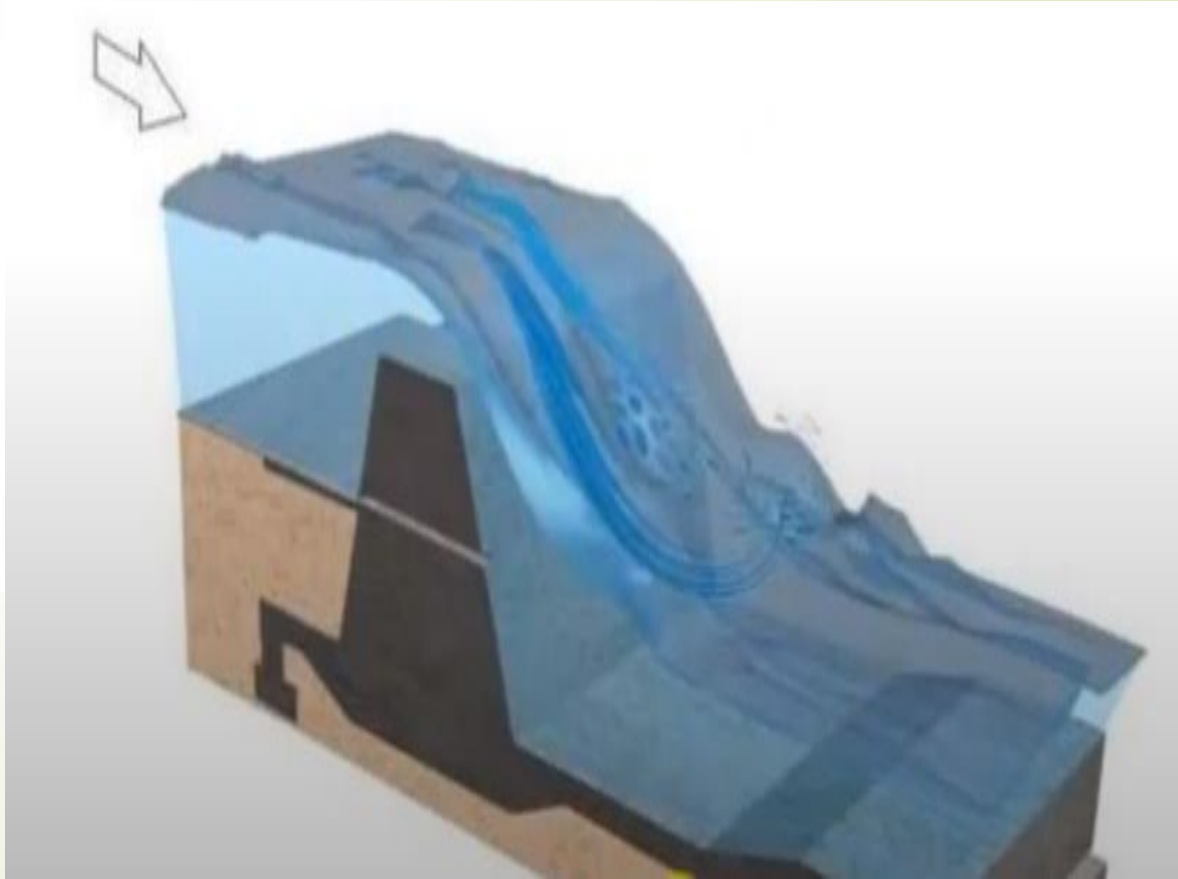
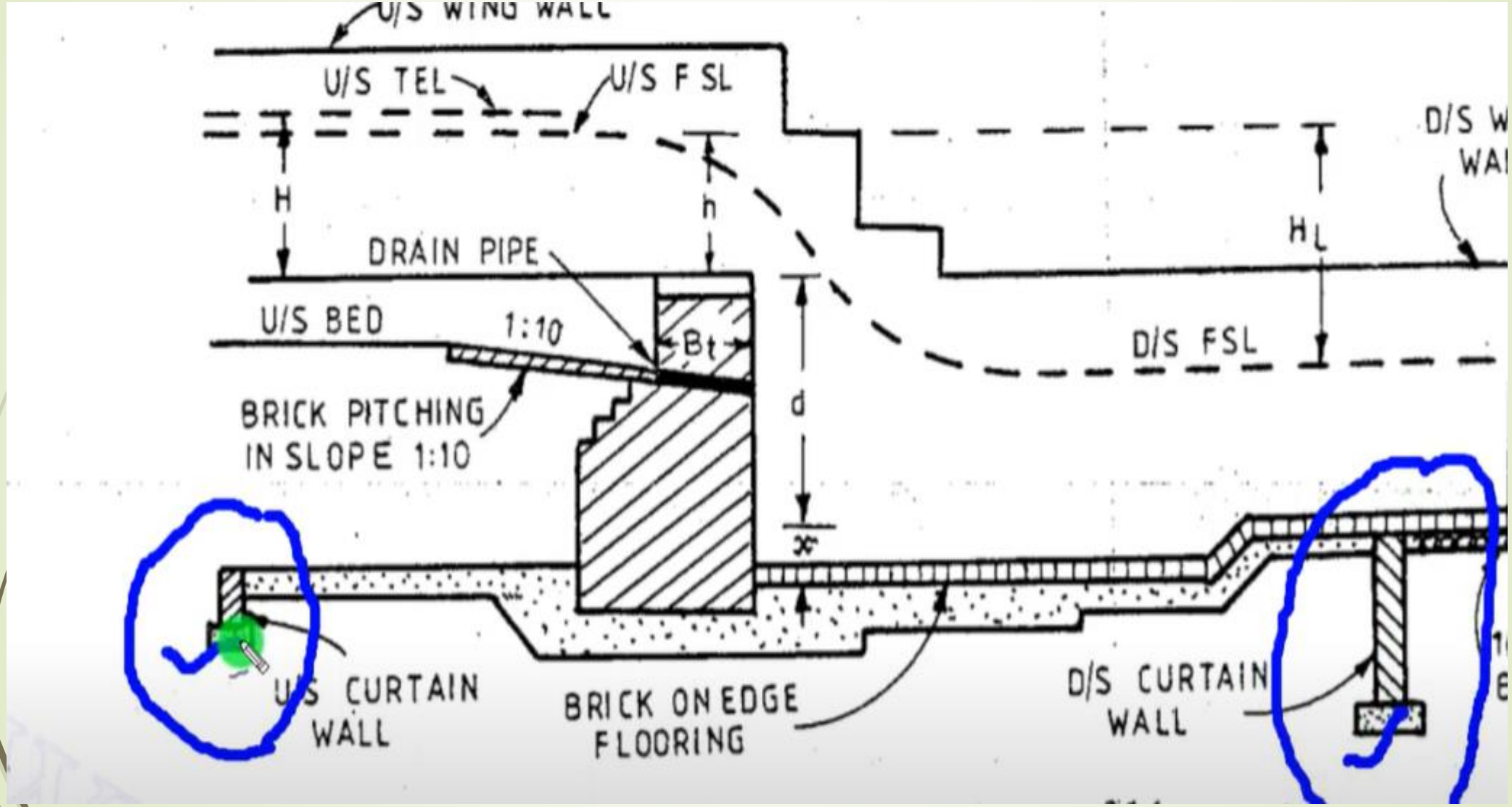


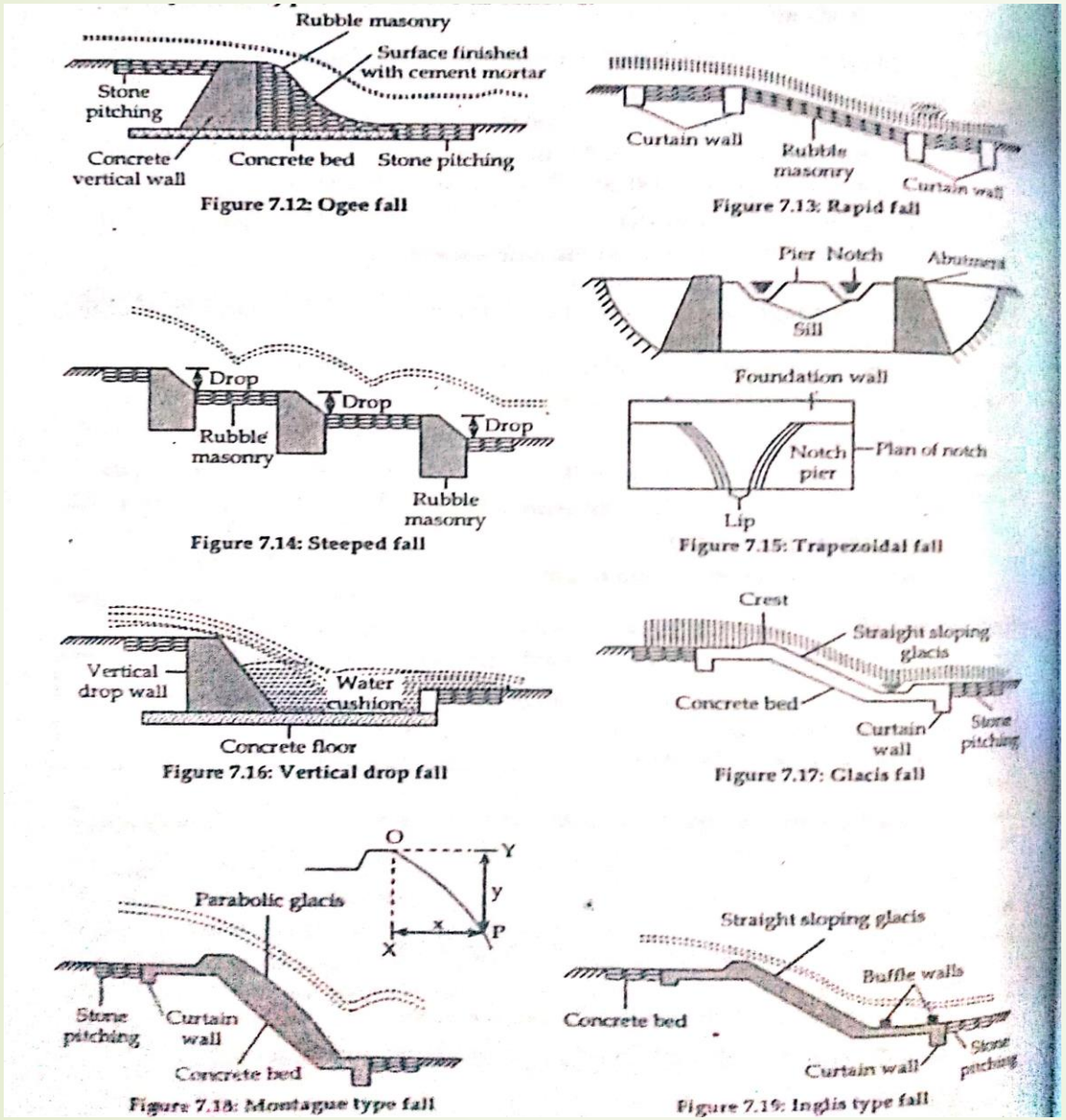
Fig: Vertical Drop Fall



Sarda fall


U/S WING WALL
U/S TEL
U/S F SL
H
h
H_L
DRAIN PIPE
U/S BED
1:10
Bt
D/S FSL
BRICK PITCHING
IN SLOPE 1:10
d
x
U/S CURTAIN
WALL
BRICK ON EDGE
FLOORING
D/S CURTAIN
WALL

❖ Types of Falls



❖ Design of Vertical Fall (Sarada type fall)

Length of crest

- Bed width of canal
- For future expansion (bed width + depth of water)

Shape of crest

Q = up to a maximum of 14 cumecs

Provide Rectangular crest

- Top width of crest (B_t) = 0.55 $\sqrt{d}$ (maximum)
- Base width = $\frac{h+d}{G}$ (minimum)
- Where, d = Height of the crest above d/s bed level.
- h = Head over the crest.
- Take, G = 2 for masonry.

For Discharge (Q) > 14 cumecs

- Provide Trapezoidal crest with u/s side slope of 1:3 and d/s slope of 1:8
- Top width of crest (B_t) = 0.55 $\sqrt{H+d}$
- Base width is calculated from side slope of crest wall.

Crest level

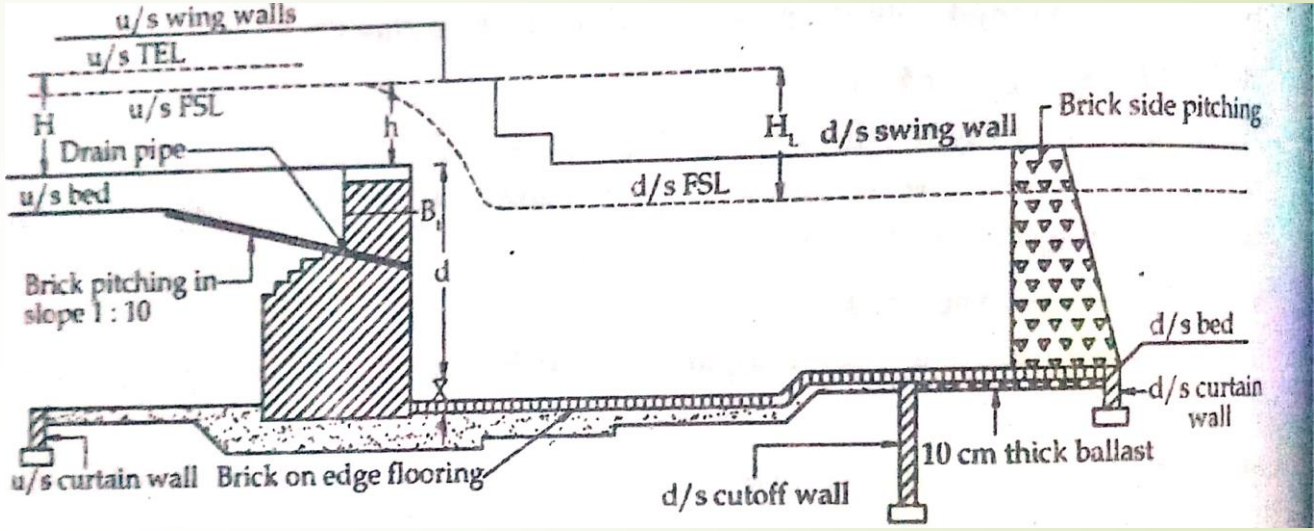
- The following formula is used to determine height of water level above the crest,
- $Q = C_d \sqrt{2g}\ L\ H^{\frac{3}{2}}\ (\frac{H}{B_t})^{\frac{1}{6}}$
- Where, C_d = 0.415 for rectangular crest.
- 0.45 for trapezoidal crest.
- L is the length of crest.
- B_t is the top width of the crest

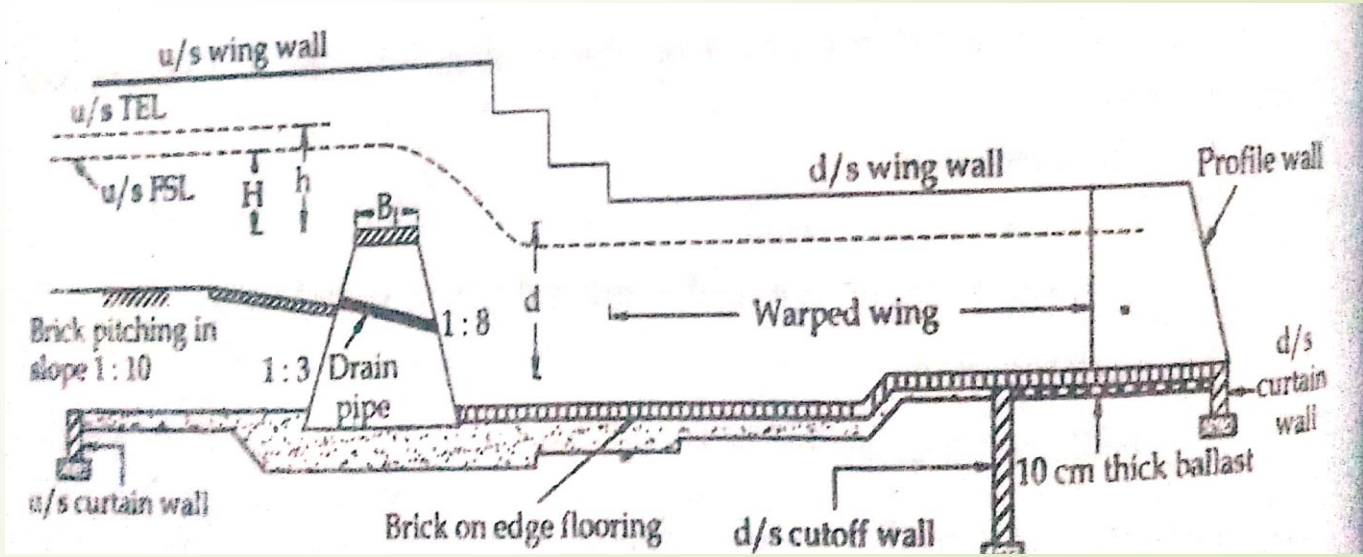
Q = up to a maximum of 14 cumecs

- $B_t = 0.55 \sqrt{d}$
- Base width = $\frac{h+d}{G}$
- $Q = 1.84\ L\ H^{\frac{3}{2}}\ (\frac{H}{B_t})^{\frac{1}{6}}$

Q = For 14 cumecs and over

- $B_t = 0.55 \sqrt{H+d}$
- Base width = $\frac{h+d}{G}$
- $Q = 1.99\ L\ H^{\frac{3}{2}}\ (\frac{H}{B_t})^{\frac{1}{6}}$





Q.N. Design vertical fall or Sarada type fall (crest, length and thickness of impervious floor) structure for the data given below.

Discharge = 1.8 m^3/sec

Bed level u/s = 205.05 m

Side slope of channel = 1:1

Bed level d/s = 204.35 m

FSL u/s = 205.95 m

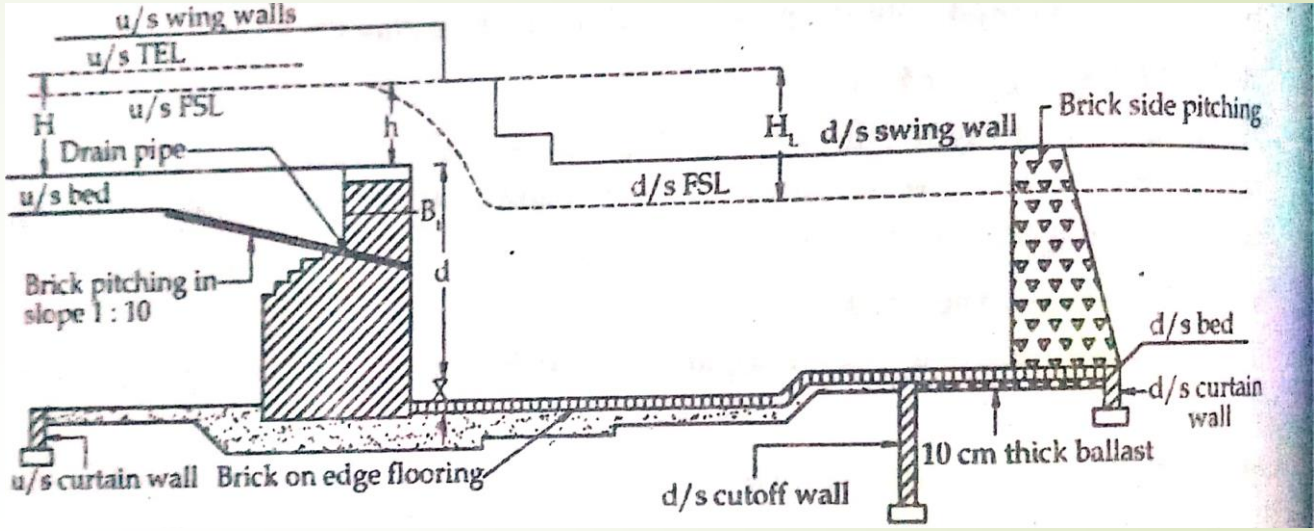
Bed width u/s and d/s = 1.5 m

Top width of crest = 0.8 (for initial assumption)

C_d = 0.415

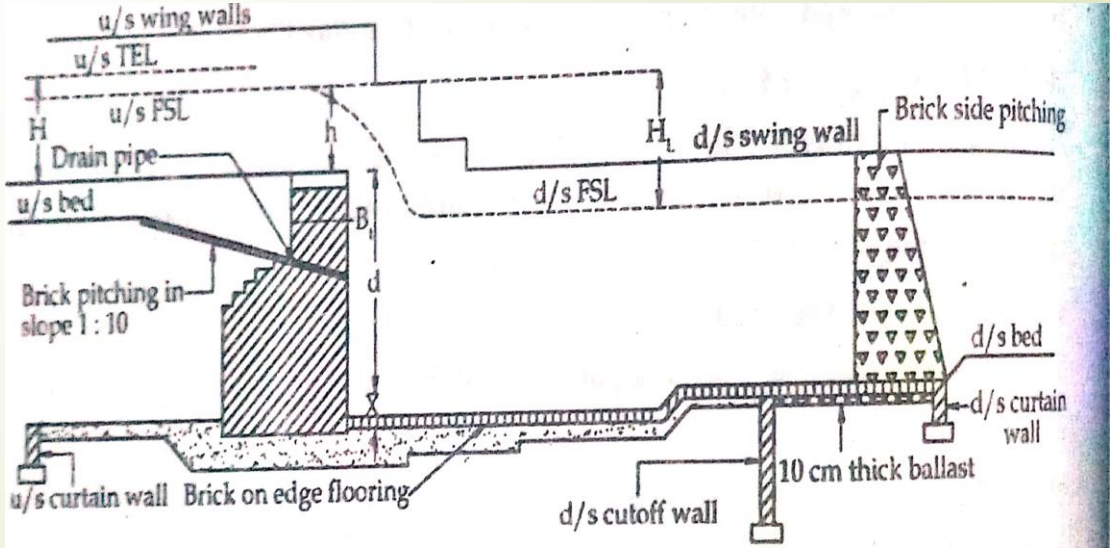
Specific gravity of masonry drop = 2

Bligh's coefficient (C) = 6.0



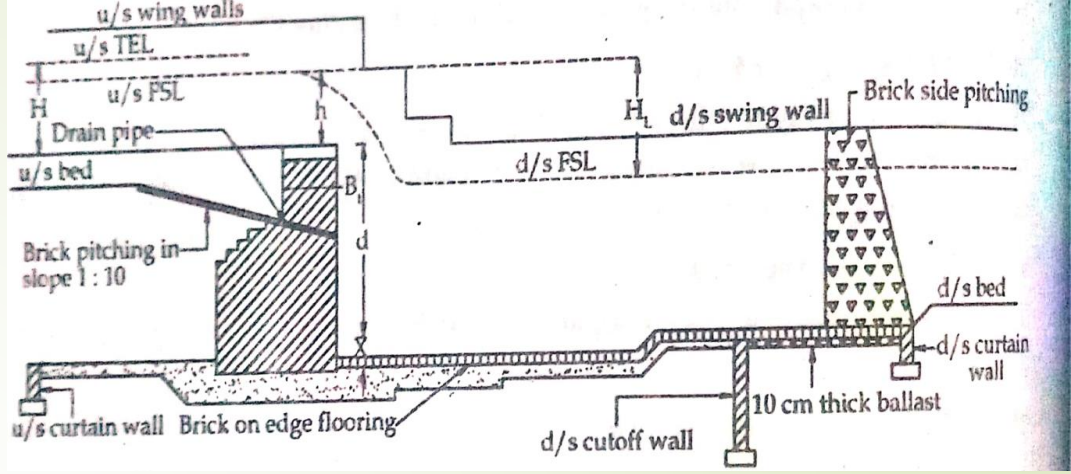
- Solution;
- Length of crest = d/s bed width = 1.5 m
- Since, Q less than 14 cumecs, rectangular crest is provided.
- Using the discharge formula; we have,
- Q = 1.84 L $H^{\frac{3}{2}}$ $(\frac{H}{B_t})^{\frac{1}{6}}$
- Let assume 0.8 m
- 1.8 = 1.84*1.5*$H^{\frac{3}{2}}$ $(\frac{H}{0.8})^{\frac{1}{6}}$
- H = 0.756 m; say 0.76 m

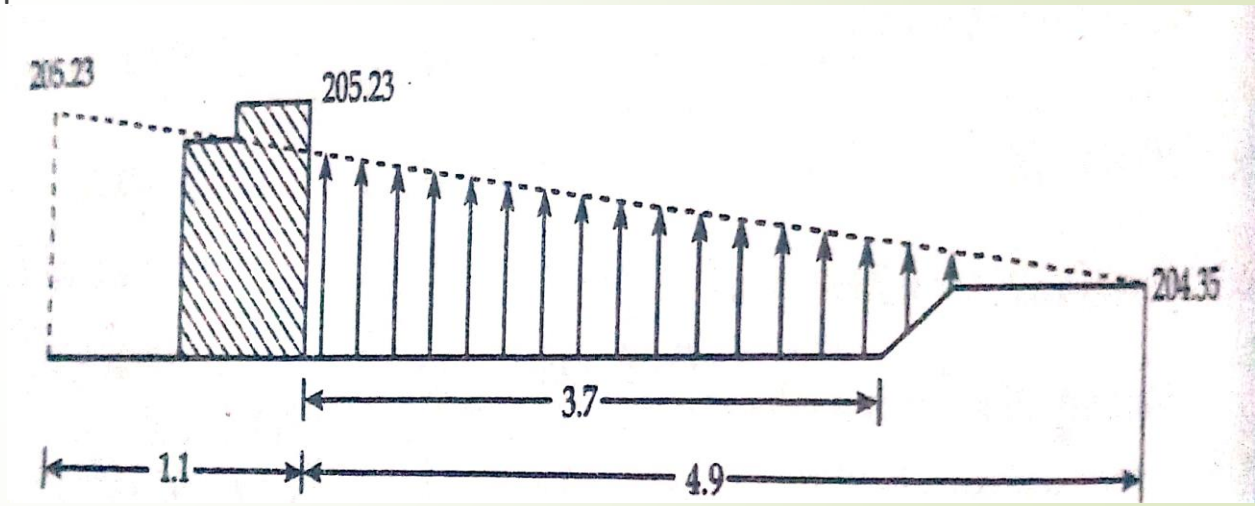
- Velocity of approach (V_a) = $\frac{discharge}{area} = \frac{Q}{(B+y)y} = \frac{1.8}{(1.5+0.9)0.9}$ (y = FSL u/s – Bed level u/s)
- V_a = 0.834 m/sec
- Velocity head = $\frac{{V_a}^2}{2g}$ = 0.035 m
- R.L of TFL = FSL u/s + Velocity head = 205.95 + 0.035 = 205.99 m
- R.L of crest = 205.99 – 0.76 = 205.23 m
- Height of crest above d/s bed = 205.23 – 204.35 = 0.88 m
- **Shape of crest**
- B_t = 0.55 $\sqrt{d}$
- d = height of crest above d/s bed = 205.23 – 204.35 = 0.88 m
- B_t = 0.55 $\sqrt{0.88}$ = 0.52 < 0.8 (OK)
- Thickness at base = $\frac{h+d}{G} = \frac{(0.76-0.035)+0.88}{2}$ = 0.81 m
- Let provide 1 m.
- Upstream curtain wall = $\frac{y_u}{3} = \frac{0.9}{3}$ = 0.3 m
- Provide 0.4*0.5 m deep curtain wall on u/s.



Cistern

- Depth of cistern (X) = $\frac{1}{4}[H * H_L]^{\frac{2}{3}} = \frac{1}{4}[0.76 * H_L]^{\frac{2}{3}}$
- H_L = u/s FSL – d/s FSL
- d/s FSL = d/s bed level + water depth = 204.35 + 0.9 = 205.25 m
- H_L = 205.95 – 205.25 = 0.7 m
- X = $\frac{1}{4}[0.76 * H_L]^{\frac{2}{3}} = \frac{1}{4}[0.76 * 0.7]^{\frac{2}{3}}$ = 0.164 use 0.2 m
- R.L of cistern = 204.35 – 0.2 = 204.15 m
- Length of cistern = 5 $\sqrt{H\ H_L}$ = 3.65 m, say 3.7m
- Provide 3.7 m long cistern at 204.15 m R.L

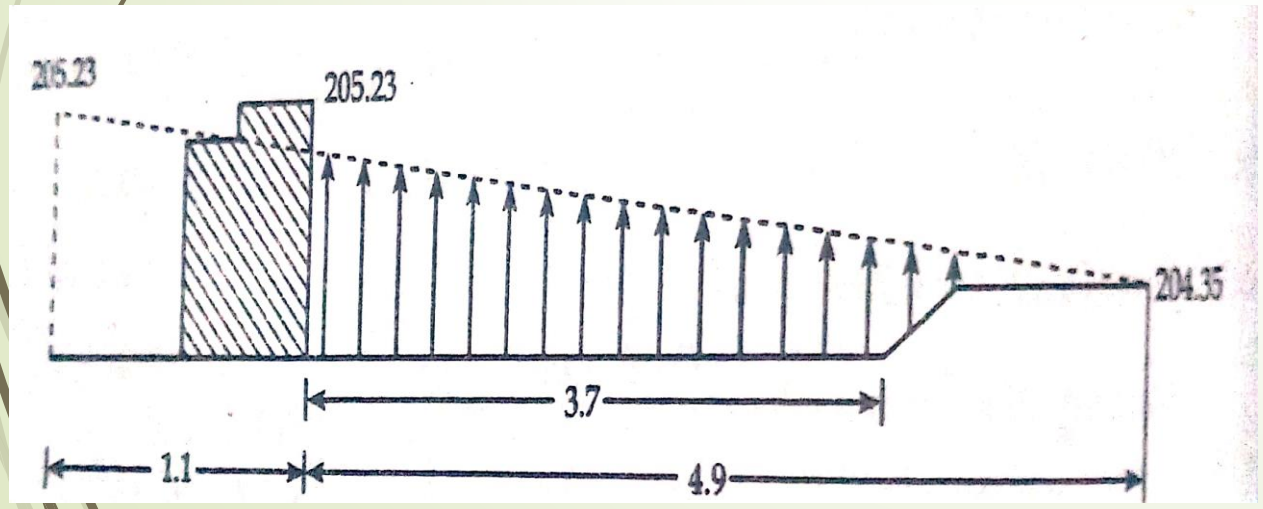


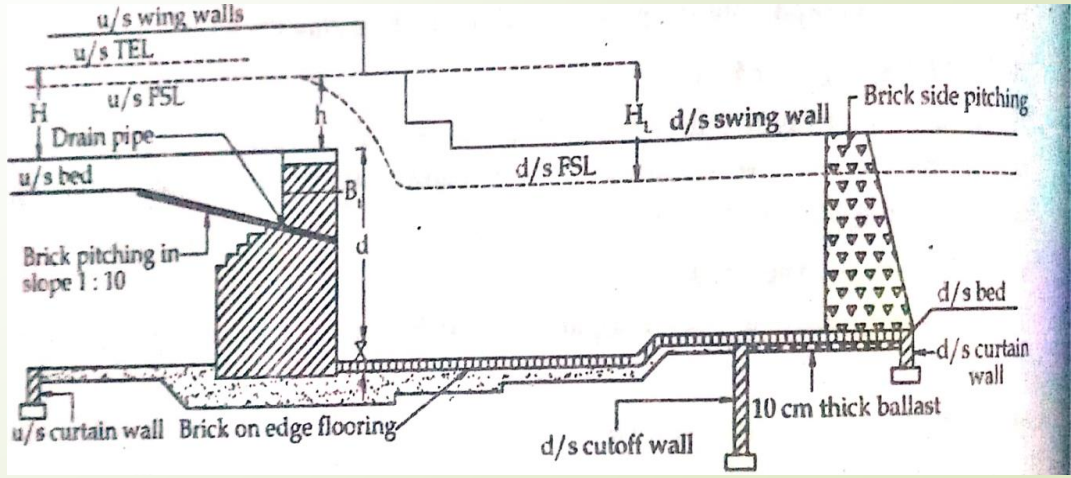


Impervious floor

- Maximum static head = crest level – d/s bed level
- H_S = 205.23 – 204.35 = 0.88m
- Total length of floor required = C H_S = 6*0.88 = 5.28 m
- Let provide 6.0 m impervious floor.
- Minimum d/s floor length required = 2[water depth + 1.2] + H_L
- = 2[0.9+1.2] + 0.7 = 4.9 m
- Provide 4.9 m d/s floor and the remaining 1.1 m on u/s.

- Maximum unbalanced uplift at d/s toe of crest = 0.2 + $\frac{H_S}{6.0}$ * 4.9
- = 0.2 + $\frac{0.88}{6.0}$ * 4.9 = 0.92 m
- Thickness required = $\frac{0.92}{G-1}$ = = $\frac{0.92}{2..0-1}$ = 0.92 m
- Provide 0.77 m thick concrete with 20 cm thick brick layer
- At 3 m from toe of crest at d/s side = 0.2 + $\frac{0.88}{6}$ * (4.9 – 3) = 0.48 m
- Thickness required = $\frac{0.48}{2.2-1}$ = 0.4 m
- Provide 0.3 m concrete with 20 cm thick brick layer and a nominal thickness of 0.3 m concrete at u/s side.





Types of Outlet, Design of outlet (Free and Submerged)

Canal outlet or Modules

- A canal outlet or a module is a small structure built at the head of the watercourse so as to connect it with a minor or a distributary channel.

- **Types of Outlets (Modules)**

1. **Non-Modular outlets**
   - Non-Modular outlets are those through which the discharge depends upon the difference of head between the distributary and the water course. The discharge through such a module, therefore, varies widely with either a change in the water level of the distributary or that of water course. Example, Open sluice.

- **Working principles of non-modular outlets;**

(i) The discharge passing through outlets depends on level difference of F.S.L. of distributary and water course and cross section area of outlet.

(ii) Required cross section of outlet for required discharge is designed according to working head H_L with formula.

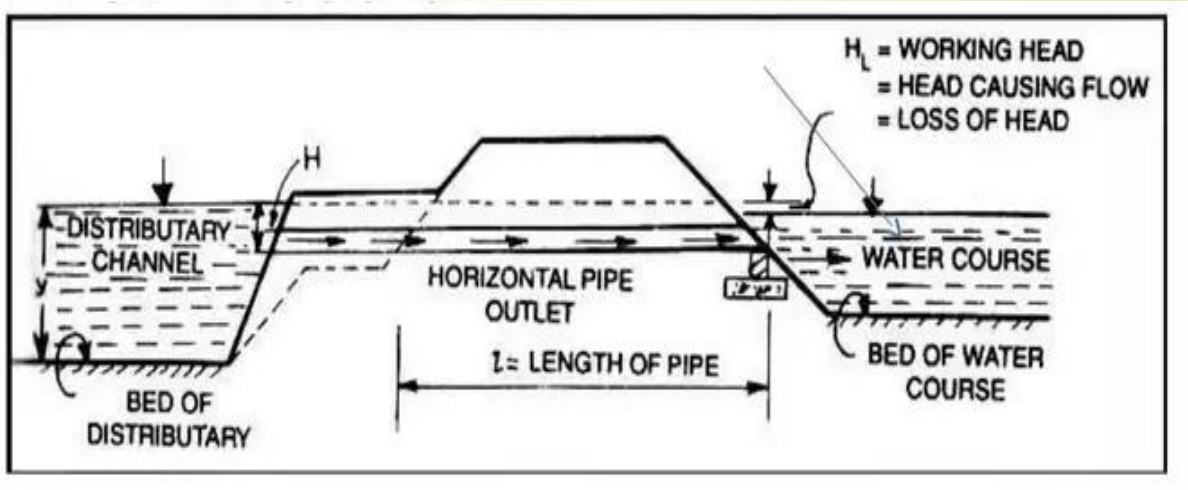
(a) For submerged pipe outlet

- $Q = C_d A \sqrt{2gH_L}$
- Cd=0.73
- Here, A is calculated.

(c) Discharge is regulated by providing a shutter on u/s end. Arrangement is provided to lock the shutter in any required position to required discharge.

- **Advantage**

- Can work in low available working head.
- Minimum head loss.
- Silt conduction is also quite good.
- Easy and simple to construct and regulate.
- Cheaper.



- **Disadvantage**

- Unequal distribution of water.
- Increase in bed level and vegetation in water course decrease discharge.

2. Semi modular or flexible modules

- These are such types of outlets in which discharge is independent of the water level of water course but depends only upon the water level of the distributary so long as the minimum working head is available. The common examples of these types of outlets are pipe outlets, venturi flume, open flume and orifice semi flume.

Advantage

- Equal distribution of water level irrespective of FSL of water course FSL.
- Free pipe outlet is simple and easy to construct.
- Silt conduction is quite good and efficiency is high.

Disadvantage

- Available level difference between distributary and water course may be insufficient.
- Except free outlet other types: semi-modular outlet is costly.

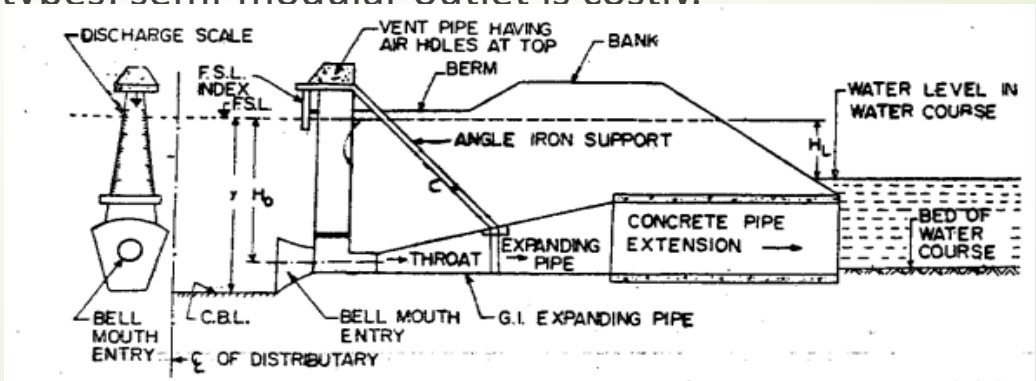


Fig. 13.15. Venturi flume outlet or Kennedy's Gauge outlet (Semi-module).

3. Rigid modules or modular outlets

- These are such types of outlet through which discharge is constant or fixed within limit, irrespective pf the fluctuation of water levels of either of distributary or of the water course or both. For examples; Gibb's module

Advantage

- Equal distribution of water level irrespective of distributary and water course
- FSL within certain working range.

Disadvantage

- It is costly and requires skill manpower to construct.

Criteria for judging the performance of Modules

- **Flexibility**

Flexibility is defined as the ratio of the rate of change of discharge of the outlet to the rate of change of the distributary channel. i.e, $F = \frac{\frac{dq}{q}}{\frac{dQ}{Q}}$

Flexibility = $F = \frac{m}{n}\frac{y}{H}$

- Where, m is the constant depending upon the type of outlet such that; $\frac{dq}{q} = m\frac{dH}{H}$
- n is the constant depending upon the type of distributary such that; $\frac{dQ}{Q} = n\frac{dy}{y}$
- y is the depth of water in distributary.
- H is the head acting on the outlet.
- **Proportionality**

The outlet is said to be proportional when rate of the change of outlet discharge is equals the rate of change of the channel discharge i.e, when flexibility is equals to 1. Hence for a proportional outlet,

$F = \frac{m}{n}\frac{y}{H} = 1$

$\frac{H}{y} = \frac{m}{n} = \frac{outlet\ index}{channel\ index}$

Sensitivity

- Sensitivity is defined as the ratio of the change of the discharge through the outlet to the rate of change of the water level of the distributary, referred to the normal depth of the channel. i.e,

- $S = \frac{\frac{dq}{q}}{\frac{dy}{y}}$

For rigid modules discharge is fixed and sensitivity is zero.

Relation between flexibility and sensitivity

We have,

- Flexibility (F) = $\frac{\frac{dq}{q}}{\frac{dQ}{Q}}$

And, $\frac{dQ}{Q} = n\frac{dy}{y}$

- $F = \frac{\frac{dq}{q}}{\frac{n}{y}dy} = \frac{1}{n}\frac{\frac{dq}{q}}{\frac{dy}{y}} = \frac{S}{n}$

- $S = n F$

Design an irrigation outlet for the following data.

F.S.Q. of outlet = 60 lit/sec

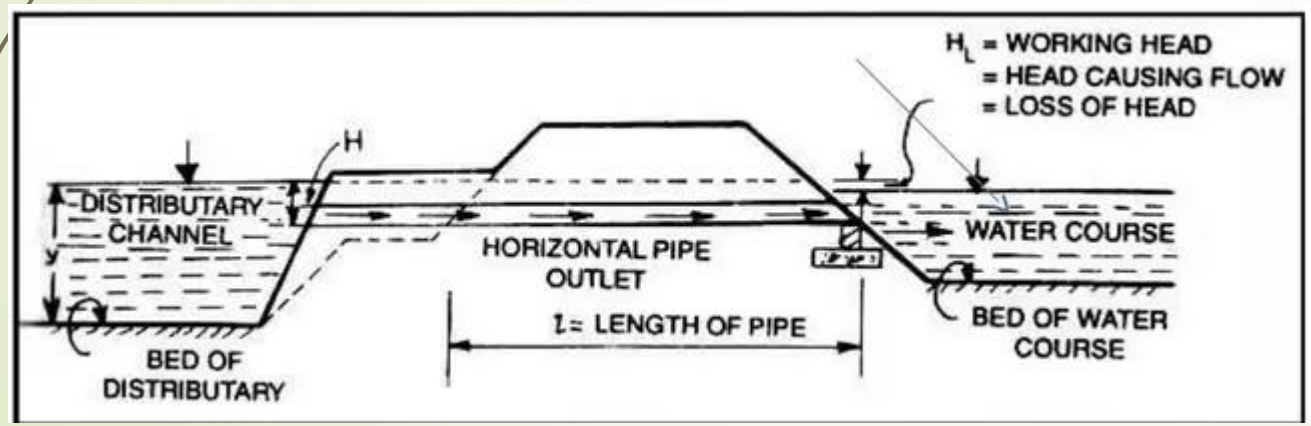
F.S.L. of distributary on u/s side of outlet = 200.00 m

F.S.L. in water course on d/s side of outlet = 199.92 m

Depth in distributary on u/s side of outlet = 1.05 m

Solution:

- Available head across outlets = F.S.L. of distributary – F.S.L. of water course
- = 200.00 – 199.92 = 0.08 m
- Since the available head is very small, a non-modular outlet (such as submerged pipe outlet) will have to be provided.
- Q = 60 lit/sec = 0.06 m^3/sec



And C_d = 0.73

Now,

- Discharge in such submerged pipe outlets
  Q = C_d A $\sqrt{2gH_L}$
- 0.06 = 0.73 A $\sqrt{2 * 9.81 * 0.08}$
- $\frac{\pi d^2}{4}$ = 0.063
- d = 0.29 m
- Here, use a pipe of diameter 30 cm, say
- The R.L. of the bed of the distributary = 200.00 – 1.05 = 198.95 m
- Let, the top of the pipe be at the level 22 cm below the F.S.L. of the distributary.
- Lower level of the pipe = 200.00 - 0.22 – 0.3 = 199.484 m

7.4 Types of outlet and design of pipe outlet

A canal outlet or a module is a small structure built at the head of the water course so as to connect it with minor or distributary canal. These outlets helps in controlling flow of water to the water course to irrigate the field.

Type of outlet:

i) Non modular outlet-Those outlets through which discharge depends upon the difference of head between the distributary and the water course. Ex: open sluice, submerged pipe outlet. They work in small head as well.

ii) Semi modular or flexible outlet-Those outlets through which the discharge is independent of the water level of the water course but depends upon the water level on the distributary as long as minimum working head is available. Ex: free pipe outlet, venture flume, orifice semi module, etc.

iii) Rigid or modular outlets- Those outlets through which discharge is constant and fixed within limits irrespective of fluctuations of the water level of either the distributary or water course. Ex: Gibb's module.

Design of submerged pipe outlet

Pipe dia. varies from 10 to 30 cm. Pipe are generally laid horizontal at about 22cm below the water level in distributary canal.

H_L=Total head loss across the outlet.

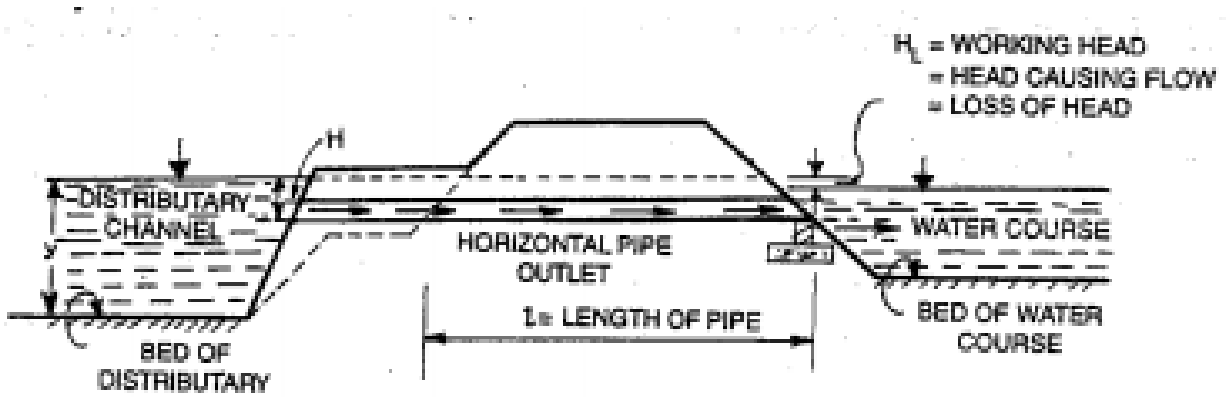
=entry loss +friction loss+ exit loss

$$= 0.5\frac{V^2}{2g} + \frac{f1lV^2}{2gd} + \frac{V^2}{2g}$$

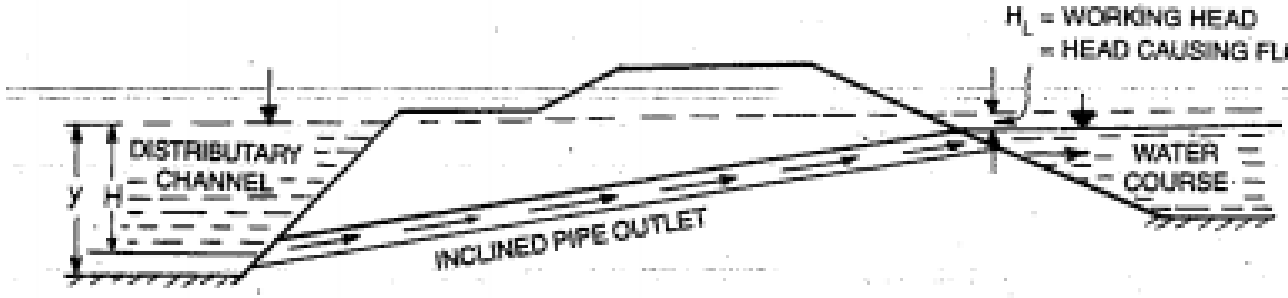
l=length of pipe

d=diameter of pipe

f1=coefficient of friction of pipe material



(a) Horizontal pipe outlet (Submerged)



(b) Inclined pipe outlet (Submerged)

Fig. 13.14. Non-modular Outlets.

‘Q’ is computed from the following equation:

$$Q = C_d A\sqrt{2gH_L}$$

C_d=coefficient of discharge=0.73

<u>Numerical:</u>

Design an irrigation outlet for the following data:

FSQ of outlet= 50 l/s

FSL in distributary=200m

FSL in water course=199.92m

Full supply depth in distributary=1.05m

Soln:

Available head (H_L) = FSL in distributary - FSL in outlet

$= 200 - 199.92 = 0.08\,m$

Since head is small non modular outlet (submerged pipe outlet) will be suitable.

$$Q = C_d \times A \times \sqrt{2gH_L}$$

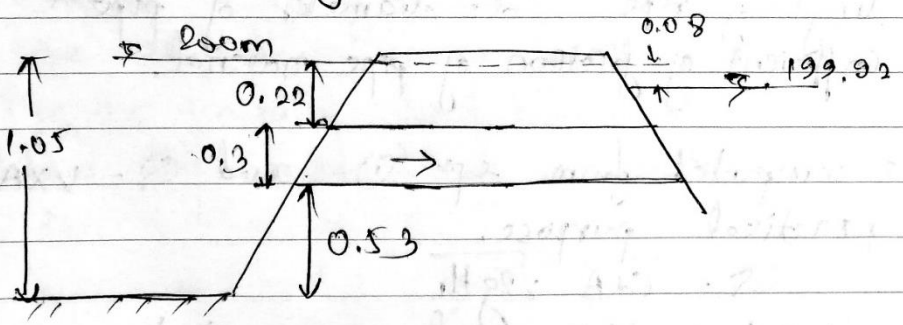
$$50 \times 10^{-3} = 0.73 \times \frac{\pi}{4} d^2 \times \sqrt{2 \times 9.81 \times 0.08}$$

$\therefore d = 0.264\,m \simeq 0.3\,m$

Use pipe of dia 30 cm.

RL of distributary bed $= 200 - 1.05 = 198.95\,m$

The pipe top is fixed 22 cm below FSL of the distributary



Design of free pipe outlet

Pipe outlet discharging freely into the atmosphere is the simplest type of flexible outlet. The discharge through the outlet depends only on the water level in distributary. Sufficient level difference between the distributary and water course is required.

Discharge eqn: $Q = C_d A\sqrt{2gH_0}$

C_d=coefficient of discharge =0.62

H_0=Head on distributary side measured from FSL to the center line of pipe.

A= area of cross section

Numerical

Design a pipe outlet for the following:

FSL in distributary=205m

FSQ at the head of water course=90 l/sec

Full supply level in water course = 204m

Soln:

Available head = 205 − 204 = 1 m

1 m of level difference is sufficient to make the pipe outlet discharge freely into water course

Assume pipe of dia 25 cm.

$$Q = C_d \cdot A \cdot \sqrt{2gH_0}$$

$$90 \times 10^{-3} = 0.62 \times \frac{\pi}{4} \times (0.25)^2 \times \sqrt{2 \times 9.81 \times H_0}$$

$$\therefore H_0 = 0.44 \text{ m}.$$

RL of centre line of pipe = 205 − 0.44
= 204.56 m

RL of outlet pipe invert = $204.56 - \frac{0.25}{2}$

= 204.43 m ↑FSL at water course

Hence, design is OK.